THIRD LAKE WASHINGTON BRIDGE

Design Criteria - Floating Structure - Stage II Pontoons

Loads

D	=	Dead Load (Includes anchor cable initial force)
H	=	Hydrostatic Pressure (at still water draft)
L	=	Live Load (Highway or Rapid Transit Alternate)
I	=	Live Load Impact
WN	=	Normal Wind on Structure - 1 Year Storm
NW	=	Normal Wave - 1 Year Storm
WS	=	Storm Wind on Structure - 100 Year Storm
SW	=	Storm Wave - 100 Year Storm
WL	=	Wind on Live Load
LF	=	Longitudinal Force from Live Load
S	=	Shrinkage and Creep
T	=	Temperature
K	=	Change in Lake Level
DM	=	Potential Damage

Basic Structural System

The pontoons are to be designed and detailed for extensive precasting (similar to Hood Canal Unit I plans) and are to be of cellular construction. The main reinforcement shall be prestressing strands and/or tendons. Stage II pontoons (except the joints at each end of Stage I) shall be connected at the joints by prestressing tendons. The joint will be equal in strength to the remainder of the pontoon.

The superstructure shall consist of composite steel girders supported by reinforced concrete columns. The columns shall be located at a transverse pontoon wall.

Vertically and torsionally the structure is considered as a continuous beam on an elastic foundation. Horizontally the structure is considered as a continuous beam with elastic supports (cables anchored to the lake bottom).

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Load Combinations

Service Load Combinations

Group	Loads	% of Basic Allowable Stress
S1	D+H+L+I+K	100
S3	Gr.S1+WN+NW+WL+LF	125
S4	Gr.S1+T+S	125
S6	Gr.S3+T+S	140
S7	Gr.S1+DM	140
S9	Gr.S3+DM	150

Factored Load Combinations

Group	Loads
U1	1.3 [D+5/3(L+I)+H+K]
U2	1.3 [D+H+WS+SW+K]
U3	1.3 [D+L+H+WN+NW+WL+LF+K]
U4	1.3 [D+(L+I)+H+K+T+S]
U5	Group U2 + 1.3 [T+S]
U6	Group U3 + 1.3 [T+S]
U7	1.3 [D+L+H+K+DM]
U8	0.92 [Group U2] + 1.2 DM
U9	0.92 [Group U3] + 1.2 DM

Design of Non-Prestressed Members

The design of non-prestressed members shall be based on behavior at service conditions (Allowable Stress Design) as per 1977 AASHTO specifications and interims through 1981, except as modified herein:

Sections where reinforcement is to resist sustained hydrostatic forces, fs shall not exceed 14,000 psi.

Design of Prestressed Members

Pre-stressed members shall be designed under the appropriate service load provisions in the AASHTO Code (Section 6) as modified herein:

For service group loadings <u>not containing</u> damaged condition:

The allowable concrete tensile stress under final conditions in the pre-compressed tensile zone shall be limited to zero.

For service group loadings <u>containing</u> damaged condition:

1. Transversely, the allowable concrete tensile stress shall be limited to $3\sqrt{f'c}$ (effect of "percent of basic allowable stess" not allowed) with mild reinforcement carrying this stress.

2. Longitudinally, the allowable concrete tensile stress shall be limited to zero, however, concrete tensile stresses up to $3\sqrt{f'c}$ may be resisted by mild reinforcement when considering "local" bending stresses (in bottom slab and webs).

When grade 60 reinforcement is used to carry the total tensile force in sections resisting sustained hydrostatic forces, fs shall not exceed 14,000 psi.

The ultimate flexural strength of the overall pontoon section computed in accordance with Section 6 of the AASHTO Code shall not be less than loads from the Factored Load Combinations except when Mcrack $>$ 4/3 Mu, then provide (Mu) cap $\geq$ 4/3 Mu.

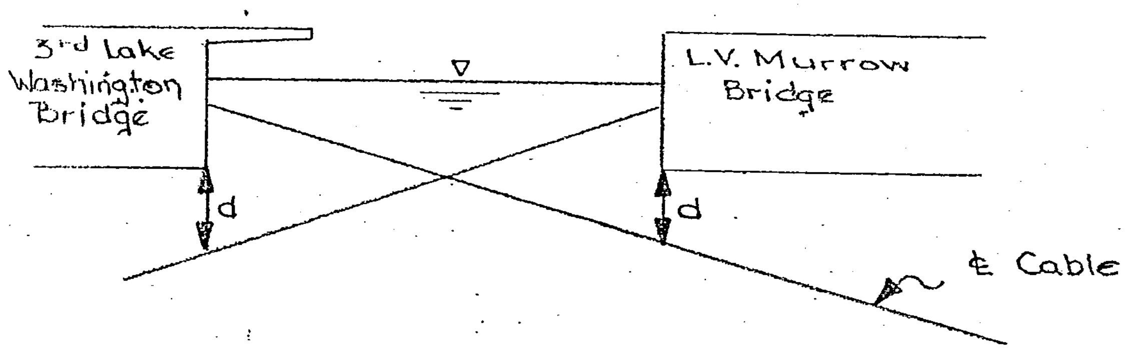
Anchorage System

The anchorage system shall be designed for the above Service Load Combinations at 100 percent of basic allowable stresses. In addition, the anchorage system shall be designed for D+H+WS+SW+K and for D+H+WS+SW+K+DM at 100 percent of basic allowable stresses. The anchor cables shall have a safety factor of 2.0 minimum (S.F. = Breaking Strength/Applied Load) during the one year storm and 1.5 during the 100-year storm (assume no broken cables or severed pontoons). The anchors shall have a factor of safety of 2.0 against failure. The anchorage system includes the anchors, the anchor cables, and the cable attachments to the pontoons.

See Final Design Notes, page 17

Anchor Cable Geometry

Clearances between an existing or new anchor cable and an adjacent new or existing pontoon bottom shall be as shown below:

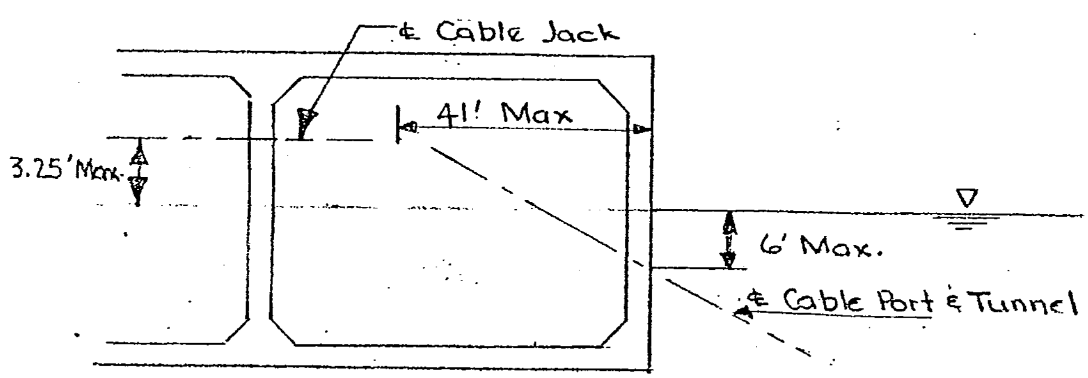


d = 1.5 foot minimum for normal conditions using an initial horizontal cable tension T_H = 120 kips in the 3rd Lake Washington Bridge and T_H = 60 kips in the L. V. Murrow Bridge.

d=1.0 foot minimum for extreme conditions using T_H = 360 kips.

Add 0.3 foot to pontoon drafts shown on L. V. Murrow Bridge plans to account for construction tolerances.

Use catenary formulas such as those given in the U.S. Steel Wire Rope Handbook to calculate the anchor cable shape. For these equations, use the horizontal and vertical distances from the anchor pin to the cable saddle. Stage II cable saddle locations may be shifted northward so as to lower the cable as it leaves the pontoon. Limitations are as shown below:



Where cables intersect with each other use a min. clearance of 2.0' under normal conditions and 1.5' under extreme conditions.

Anchor pin evaluations can be determined by the following criteria:

Type A-J Anchor	-	Subtract 10' from lake bottom elevation.
Type A-EB Anchor	-	Subtract 8' from lake bottom elevation.
Type B Anchor	-	Add 1' to lake bottom elevation.
Type C Anchor	-	Add 1' to lake bottom elevation.

<u>End Walls</u>

Design end transverse walls (end bulkheads) to resist hydrostatic pressure full height of the wall. In addition, design end bulkheads to withstand grout pressure for pour rate of 4 feet per hour at 55^{o}F (800 P.S.F.), see Std. Specs. 6-02.3(17)F). Design by Service Load methods.

<u>Interior Walls</u>

Interior walls shall be designed for the above group loadings and for flooding on one side. For flooding on one side, design transverse walls to withstand a 100 percent impact due to rushing water at the equilibrium water level and to withstand full height flooding without impact. For flooding on one side, design longitudinal walls to withstand flooding at equilibrium water level without impact and utilizing load factor methods.

Transverse anchor walls shall be designed and detailed as cast-in-place by service load methods.

<u>Material Properties</u>

Concrete:

The compressive strength (28 day) of the concrete shall be established by analysis but shall not be <u>less</u> than 5,000 psi nor more than 6,500 psi. $Ec=W^{3/2}33$ fc.

Post-Tensioning Tendons:

Strands - ASTM A416 Grade 270, E_s=28,000 ksi
(½" dia. or 0.6" dia. strands may be used)

Bars - ASTM A722 E_s=28,000 ksi

Mild Steel Reinforcement:

ASTM A615 Grade 60

Grout:

The compressive strength of all grout used in pre-stressing tendons and the pontoon to pontoon connection shall be equivalent to that defined above for concrete.

Anchor Cables - Galvanized Bridge Strand:

Diameter = 2 3/8 inches
Breaking Strength = 668 kips
Steel Area = 3.38 in^2
Weight = 11.85 lbs./ft.
E_s = 23,000 ksi

Description of Loads

Dead Loads

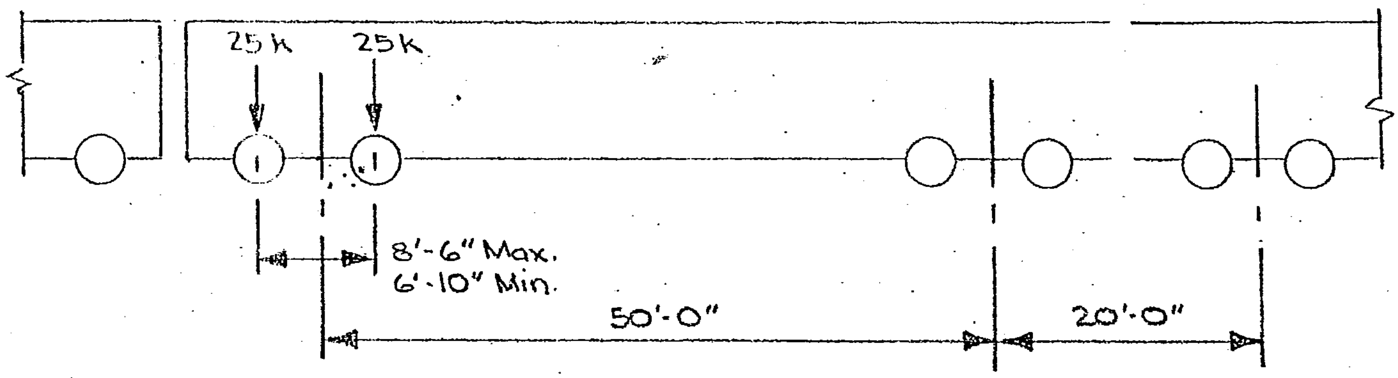
Reinforced concrete unit weight shall be taken initially as 160 lb./cu. ft. (including reinf.) for computing dead load stresses and draft. As the design progresses, adjustments will be made based on a concrete weight of 152 #/c.f. plus actual weight of the reinforcing steel and prestressing steel required.

The vertical component of initial cable load may be considered as a dead load. Variations in the cable load due to displacements from other loadings will be included in the effect of each loading, if significant.

Live Load

HS 20 Truck or Lane Loading shall be used without modification. Military Loading of 2-24 kip axles at 4' centers shall also be considered.

The reversible lanes shall also be designed for the Rapid Transit loading of the Puget Sound Governmental Conference Rail Rapid Transit Design Criteria shown below.



Car Length	70'
Car Height	12' Max
Car Width	10'
Speed on Bridge	45 mph
Axle Load	25 kips
Impact	$\frac{100\ LL}{DL + LL}$ (30% Max)
Lane Width	14' Min.
Traction Force	15%
Wind	300 lbs per lin. ft. of train
Rail Weight	100 lbs per yard
Acceleration and Deceleration Rate	3.5 mph/sec Max.
Number of Cars per Train Unit	8 Max.
Number of Trains on Bridge at Same Time	2 - 1 in each direction
Allowable Grade	5% Max.

Rapid Transit Loading shall be used in combination with the Highway Loadings, considering each track of Rapid Transit as a lane for use of the multiple lane reduction factor.

In addition to Section 1.2.9 of AASHTO Specifications, 60% of the resultant live load stresses shall be used when produced by loading 6 or more traffic lanes simultaneously. (Ontario Code)

Under towing and construction conditions, the top slab of roadway pontoons shall be adequate to take an H-10 loading, or to take 200 psf. loading.

Under the elevated roadway, the top slab of the pontoon shall be designed for a single H-10 maintenance truck, or to take 200 psf. loading.

Live Load Impact

Impact shall be applied in computing local stresses in the roadway slabs and superstructure only, not for overall pontoon stresses.

Wind and Wave - General

Wind blowing over water generates a sea state that includes horizontal, vertical and torsional loads on the bridge structure. These loads are a function of the wind velocity, wind direction, wind duration, fetch (distance over water along which wind blows) and the channel configuration and depth. The bridge shall be designed for the combined effect of wind and wind-driven waves, both perpendicular and skewed to its longitudinal axis and assuming the First Lake Washington Bridge is not in place. Consideration shall be given to the normal storm and extreme storm wind and wave conditions as indicated in the load combinations. The normal storm conditions are defined as the storm conditions that have a recurrence interval of 1 year (i.e., the maximum storm that is likely to occur once a year). The extreme storm conditions are defined as the storm conditions that have a recurrence interval of 100 years.

Wind Force on Structure - 100 Year Storm

See Final Design Notes

The overall wind force to be resisted by the pontoon shall be 30 psf. applied normal to the longitudinal axis of the bridge.

Individual members shall be designed per AASHTO wind loads.

A separate longitudinal wind force of 28% of the transverse force shall be used.

An upward overturning force shall be applied to the superstructure per AASHTO Specifications.

Wind Force on Live Load - 100 Year Storm

Use 60 percent of the AASHTO Specifications values for highway loading. For Rapid Transit Loading use 25 psf on the train area exposed above the rail base.

Wind Force on Structure - 1 Year Storm

The wind loads generated by the 1 year storm shall be considered equivalent to 30 percent of the wind loads generated by the 100 year storm. (Consistent with Hood Canal criteria.)

Wave Forces on Structure - 100 Year Storm

Wave forces shall be determined by the criteria included in Appendix A and using the following design wave characteristics:

Significant wave height	=	4.8' ← See Final Design Notes
Maximum wave height	=	8.0'
Significant wave period	=	4.6 seconds
Wave length	=	60'
Wave crest length (parallel to the Bridge)	=	420'

This data is consistent with Stage I design. The data prepared by Prof. Reed is somewhat less than the above and will be utilized in the dynamic analysis.

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Wave Forces on Structure - 1 Year Storm

Wave forces generated by the 1 year storm may be considered equal to 30 percent of wave loads generated by the 100 year storm.

Dynamic Forces

At the time of writing these criteria, a consulting firm is being engaged to make a dynamic analysis of the floating structure. Since this analysis will not be complete for some time, the design is to proceed using the wind and wave forces defined above. As soon as the consulting firm has determined the dynamic forces, the structure's capacity will be reviewed and adjustments made as required.

The 100 year storm data in the following table will be provided to the consultant for their analysis. These storm data were prepared by Professor Richard J. Reed and included in the Tokola/Earl & Wright Study.

Wind Direction	Wind Speed, mph.			Storm Heading (Azimuth)	Fetch in Naut. mi.	Significant wave height ft.	Significant wave period sec.
	10 sec.*	1 min.	1 hr.				
North	49	45	37	0-30°	6.5	3.04	3.62
				30°-60°	2.5	2.55	2.91
				60°-90°	negligible		
				270°-0°	negligible		
South	66	60	50	90°-120°	negligible		
				120°-180°	2.5	2.55	2.91
				180°-270°	negligible		

* 10 Second gust speeds were determined by multiplying 1 hour mean by 1.31

100 YEAR STORM CONDITIONS

Temperature

A temperature differential between various portions of the structure shall be considered as follows:

From submerged portion to exposed portion $\pm 35^{o}$
From submerged portion to shaded portion $\pm 25^{o}$ F
From shaded portion to exposed portion, $\pm 25^{o}$ F

The submerged portion is the entire pontoon except top slab. The shaded portion is the top slab of pontoon beneath the elevated roadway. The exposed portion is the roadway slab, either on the elevated roadway or on the pontoon.

The temperature differential is the difference between average temperature of the various structural portions. No reduction in concrete modulus.

Shrinkage

Differential shrinkage shall be considered, as applicable, between the different components of the pontoon. The ultimate differential shrinkage strain shall be 0.0002. The contractor shall be directed to adjust the plan length of each pontoon as necessary to compensate for shrinkage.

The effect of shrinkage shall be considered to take place in the "dry" condition only. The shrinkage rate can be assumed to be zero upon launching of the pontoon. Shrinkage of any time may be computed as outlined in A.C.I. Publication SP-27.

Elastic Shortening & Creep

The contractor shall be directed to adjust the plan length of each pontoon as necessary to compensate for elastic shortening and creep due to prestress.

The effect of creep shall be considered to take place in the "dry" condition only. Creep can be assumed to be zero upon launching of the pontoon. Creep at any time may be computed as outlined in A.C.I. Publication SP-27.

Longitudinal Force from Live Load

Per AASHTO Specifications, or Rapid Transit Criteria. In applying the AASHTO Specifications the load used shall be the HS 20 Lane Loading for 1,000 ft. of the structure, then reducing linearly to HS 15 Lane Loading in 500 feet, and the remaining structure shall be loaded with HS 15 Lane Loading. 5 lanes shall be used.

Change In Lake Level

Maximum Rise = 0.8 feet.
Maximum Fall = 3.8 feet. (Normal water level elev. = 8.02)

Earthquake

The pontoon need not be designed for earthquake generated forces. It is felt the water will provide sufficient dynamic absorption capacity to minimize any earthquake loads.

Potential Damage

Stream flow, drift and ice are considered negligible in Lake Washington.

Damage to the floating portion of the structure could occur from a collision with a boat, from severing of an anchor line or from other unforeseen accidents. Damage to the pontoon bottom slab or exterior wall could result in loss of buoyancy locally. Thus, forces arising from the loss of buoyancy of any two adjacent cells, the flooding of all cells across the width of the pontoon, the flooding of all cells adjacent to one lateral cable wall, the flooding of the four cells adjacent to the cable adjustment device at cross-pontoons, or forces from the severing of any <u>one</u> anchor line shall be considered during normal operating conditions. The complete separation of a pontoon by a transverse or diagonal fracture shall be considered under ultimate conditions only.

In addition, the flooding of the five end cells of an isolated pontoon during towing or construction shall be considered.

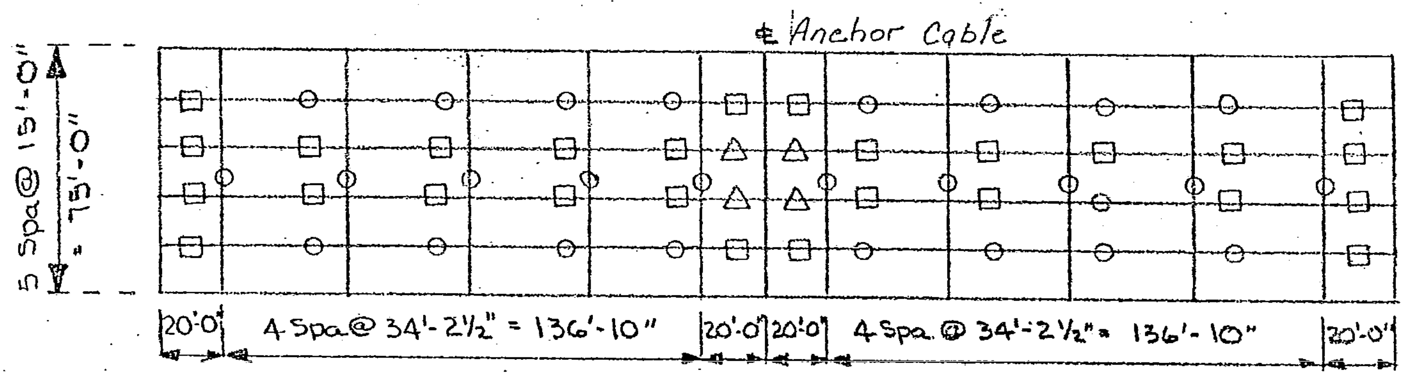
Consider only one damage condition and location at any one time.

In designing the pontoon exterior walls, apply 10 kips horizontal collision load as a service load. However, the exterior wall panel shall have an ultimate strength capacity to withstand 30 kips.

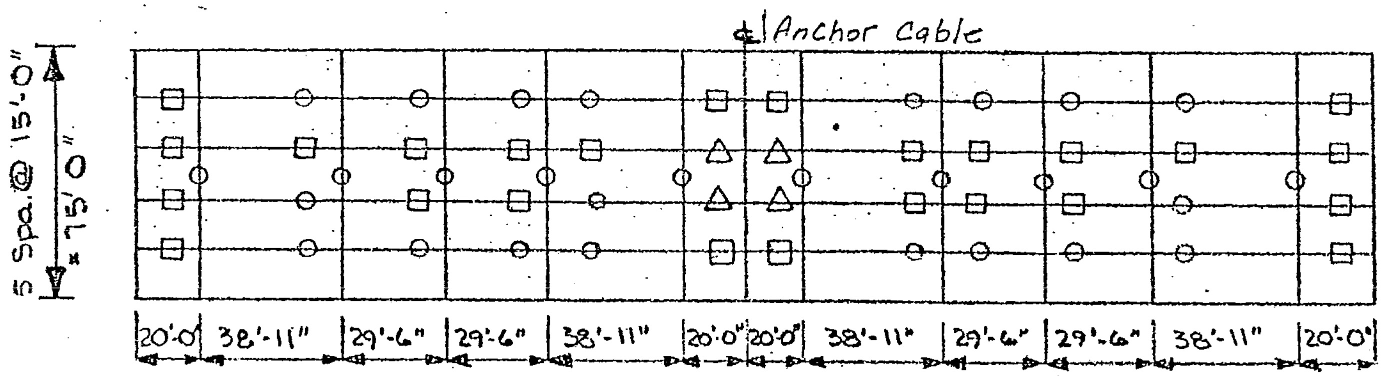
Interior Wall Location and Wall Openings

Provide sufficient interior walls, watertight doors and freeboard on wall openings to withstand potential damages previously mentioned without resulting in a progressive failure.

Stage I wall spacing and watertight door locations meeting these criteria are shown below. Stage II walls and doors are to be located in a similar manner.



Pontoons F-I & K-O



Pontoon J

□ Denotes wall opening between cells with 2'-3" freeboard

△ Denotes wall opening between cells without freeboard

○ Denotes wall opening between cells with watertight door and 1'-1" freeboard

Reinforcing Steel Cover

The cover from the face of concrete to any reinforcing bar shall be as follows:

Top of roadway slab *	1½"
Exterior surfaces of pontoons except roadway slab	1½"
All surfaces of pedestrian and traffic barriers	1½"
All other surfaces	1"

* An 1½" latex modified concrete overlay will be placed atop the roadway slab providing 3" minimum cover.

Epoxy Coated Reinforcement

The top mat of the roadway slab and the top mat of the upper pontoon slab of the elevated pontoons shall be epoxy coated.

Freeboard

The roadway pontoons shall provide a minimum freeboard of 7'6". The superstructure pontoons shall provide a freeboard of 7'-0". The freeboard shall be measured from the top of the deck at the edge of the slab to the normal water level. The freeboard shall be calculated based on the following criteria.

a) The concrete weight shall be taken as 152 p.c.f. plus the weight of the prestress and reinforcing steel involved.

b) There shall be no live load on the pontoon.

c) The vertical component of the anchor cable force shall be considered as dead load.

d) The weight of catwalk, ladders, and other hardware shall be included.

e) The unit weight of water is 62.4 p.c.f.

f) Construction Tolerances.

g) Weight of reserve ballast.

Construction Tolerances

The theoretical draft of the pontoons shall be increased 4% to accommodate possible construction inaccuracies. This 4% figure is derived from experience gained from construction of Lacey V. Murrow Bridge and Evergreen Point Bridge.

Reserve Ballast

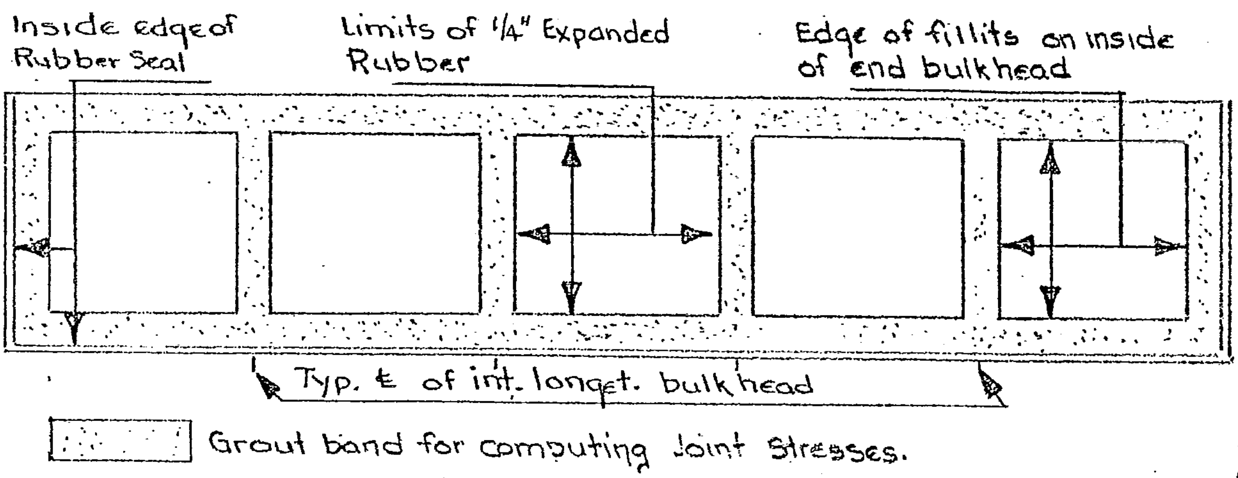
Stage II pontoons shall be designed to accommodate an amount of ballast that will displace the pontoon 3 inches vertically. Stage II pontoons shall be designed to mate with Stage I pontoons with the ballast in place. This ballast is intended to compensate for unforeseen (unplanned) increases in pontoon draft and shall be in addition to the effect described under Construction Tolerances.

Roll

The bridge comprising Stage I and II shall be analyzed as a unit to determine the magnitude of roll. Loads shall be applied as defined in the service load combinations except combinations containing the 100 year storm shall not apply. Consideration should be given to applying live loads that realistically occur on the structure.

Pontoon Joints

In computing reinforcement requirements and grout (concrete) stresses at pontoon joints AB, BC, CD, DE, PQ and QR, follow section 6 of the AASHTO code. The effective grout bearing area (grout band) of these joints is shown below.



END ELEVATION OF PONTOON

Sufficient prestress reinforcement shall be placed across the joint to provide zero tensile stress under all service load groups. In addition, the joint shall be designed for a residual compressive stress for loading combinations not containing the 100 year storm. This residual shall be equal in magnitude to 25 percent of the flexural stress resulting from the applied moment.

The bolted pontoon joints EF and OP shall be designed by the methods used for Stage I.

The temporary clamping force at pontoon joints during grouting shall be as defined in the Stage I plans, sheet 12 of 55.

Pontoon Shear Design

Shear stress taken by concrete in longitudinal walls shall be taken as $v_c = 1.7\ f_c'$. Shear stress in excess of v_c shall be resisted by reinforcement. Mild steel reinforcement or prestress reinforcement can be used with an allowable stress of $v_s = 60$ Ksi.

$$v_u = \phi (v_c + v_s) \qquad \text{A.A.S.H.T.O. 1.6.13(A)}$$

Reinforcement required for shear shall be in addition to that required for torsion, bending, and etc.

Launching Draft

At the time of writing this criteria, three construction yards are available in the Puget Sound area having the experience and capability to manufacture the Stage II pontoons. The following table summarizes each yard's launching draft capacity.

Firm	Location	Sill Elev.	Launching Draft*
General Construction Co.	Seattle	-2.0 ft.	13.0'
Concrete Technology Corp.	Tacoma	-2.0 ft.**	13.0'
J. A. Jones Co.	Tacoma	-3.5 ft.	14.5'

* At 11.0 High Tide

** Floor Elevation

Based upon data from the above table, design each pontoon for a launching draft to not exceed 13.0 feet.

Final Design Notes

The design wind pressure of 30 psf and design significant wave height of 4.8' were initially selected for these criteria because they were thought to be consistent with Stage I design and would be conservative. This conservatism was chosen in anticipation of larger forces resulting from the Dynamic Analysis.

Upon receiving the foundation report (copy attached) late in the design process, it was discovered the fluke anchor load resisting capacity was less than the maximum loads generated by 30 psf and a 4.8' wave. Therefore, design loads were recalculated for a wind speed of 66 mph and a significant wave height of 3.04'. A summary of minimum cable loads follows:

Initial Deisgn: (30 psf and 4.8' wave)
Max. Ultimate Cable Force = 401^k (Group U 6)
Max. Service Load Cable Force = 204^k (Group S 6)

Final Design Loads (66 mph wind and 3.04' wave)
Max. Ultimate Cable Force = 203^k
F.S. = 150(2)/203 = 1.5 against failure

Max. Service Load Cable Force = 145^k
F.S. = 100(2)/145 = 1.4 against proof load

It was concluded the final design loads vs capacities noted in the foundation report are acceptable with the exception that the proof load be increased from 90 tons to 100 tons for both Stage I and Stage II fluke anchors.

8/BR23